CHAPTER 13

Eccentrically Loaded Bolted Connections and Historical Notes on Rivets

13.1 BOLTS SUBJECTED TO ECCENTRIC SHEAR

Eccentrically loaded bolt groups are subjected to shears and bending moments. You might think that such situations are rare, but they are much more common than most people suspect. For instance, in a truss it is desirable to have the center of gravity of a member lined up exactly with the center of gravity of the bolts at its end connections. This feat is not quite as easy to accomplish as it may seem, and connections are often subjected to moments.

Eccentricity is quite obvious in Fig. 13.1(a), where a beam is connected to a column with a plate. In part (b) of the figure, another beam is connected to a column with a pair of web angles. It is obvious that this connection must resist some moment, because the center of gravity of the load from the beam does not coincide with the reaction from the column.

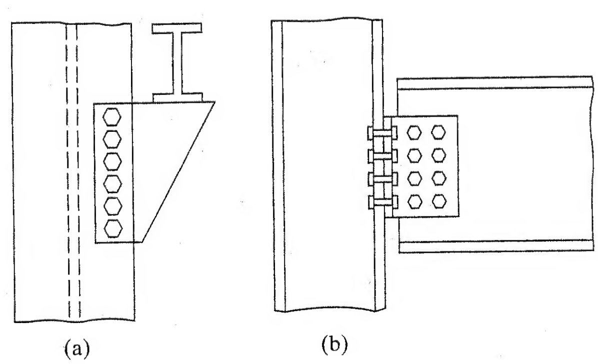
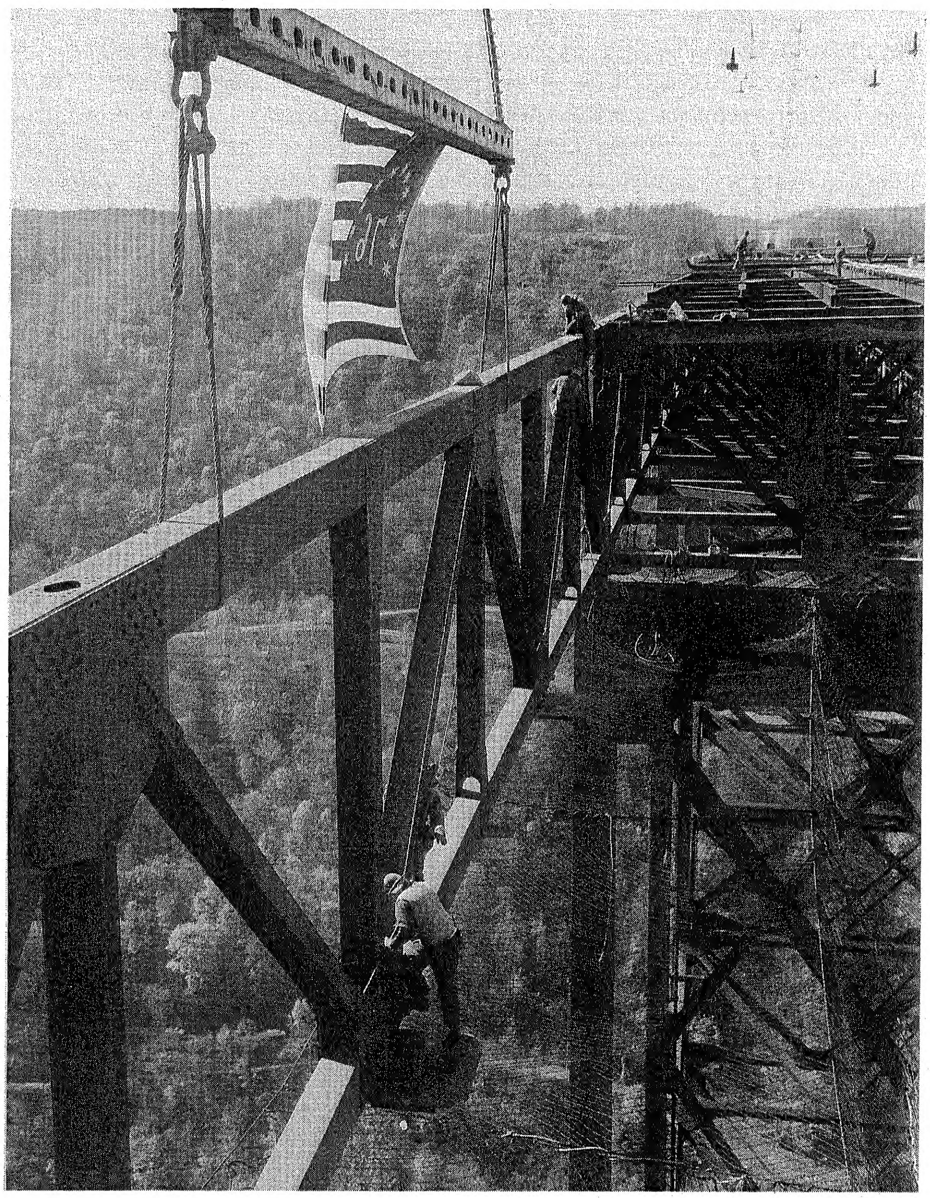


FIGURE 13.1

Eccentrically loaded bolt groups.



Bridge over New River Gorge near Charleston in Fayette County, WV. (Courtesy of the American Bridge Company.)

In general, specifications for bolts and welds clearly state that the center of gravity of the connection should coincide with the center of gravity of the member, unless the eccentricity is accounted for in the calculations. However, Section J1.7 of the AISC Specification provides some exceptions to this rule. It states that the rule is not applicable to the end connections of statically loaded single angles, double angles, and similar members. In other words, the eccentricities between the centers of gravity of these members and the centers of gravity of the connections may be ignored unless fatigue loadings are involved. *Furthermore, the eccentricity between the gravity axes and the gage lines of bolted members may be neglected for statically loaded members.*

The AISC Specification presents values for computing the design strengths of individual bolts, but does not specify a method for computing the forces on these fasteners when they are eccentrically loaded. As a result, the method of analysis to be used is left up to the designer.

Three general approaches for the analysis of eccentrically loaded connections have been developed through the years. The first of the methods is the very conservative *elastic method* in which friction or slip resistance between the connected parts is neglected. In addition, these connected parts are assumed to be perfectly rigid. This type of analysis has been commonly used since at least 1870.[1,2]

Tests have shown that the elastic method usually provides very conservative results. As a consequence, various *reduced* or *effective eccentricity methods* have been proposed.[3] The analysis is handled just as it is in the elastic method, except that smaller eccentricities, and thus smaller moments, are used in the calculations.

The third method, called the *instantaneous center of rotation method*, provides the most realistic values compared with test results, but is extremely tedious to apply, at least with handheld calculators. Tables 7-7 to 7-14 in Part 7 of the Manual for eccentrically loaded bolted connections are based on the ultimate strength method and enable us to solve most of these types of problems quite easily, as long as the bolt patterns are symmetrical. The remainder of this section is devoted to these three analysis methods.

13.1.1 Elastic Analysis

For this discussion, the bolts of Fig. 13.2(a) are assumed to be subjected to a load P that has an eccentricity of e from the c.g. (center of gravity) of the bolt group. To consider the force situation in the bolts, an upward and downward force—each equal to P—is assumed to act at the c.g. of the bolt group. This situation, shown in part (b) of the figure, in no way changes the bolt forces. The force in a particular bolt will, therefore, equal P divided by the number of bolts in the group, as seen in part (c), plus the force due to the moment caused by the couple, shown in part (d) of the figure.

The magnitude of the forces in the bolts due to the moment Pe will now be considered. The distances of each bolt from the c.g. of the group are represented by the values d_1, d_2, etc., in Fig. 13.3. The moment produced by the couple is assumed to cause the plate to rotate about the c.g. of the bolt connection, with the amount of rotation or

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[1]W. McGuire, *Steel Structures* (Englewood Cliffs, NJ: Prentice-Hall, 1968), p. 813.

[2]C. Reilly, "Studies of Iron Girder Bridges," *Proc. Inst. Civil Engrs.* 29 (London, 1870).

[3]T.R. Higgins, "New Formulas for Fasteners Loaded Off Center," *Engr. News Record* (May 21, 1964).

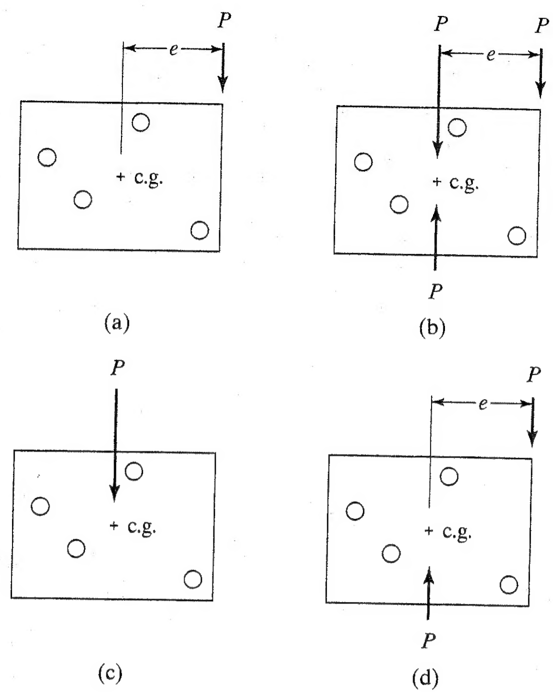


FIGURE 13.2

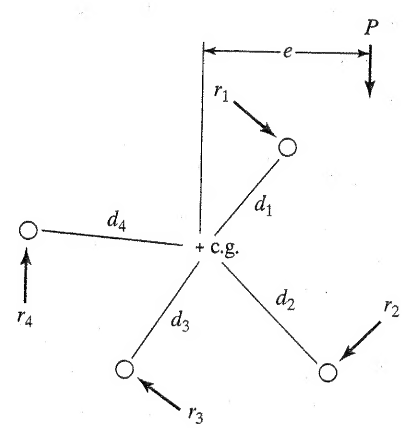


FIGURE 13.3

strain at a particular bolt being proportional to its distance from the c.g. (For this derivation, the gusset plates are again assumed to be perfectly rigid and the bolts are assumed to be perfectly elastic.) Stress is greatest at the bolt that is the greatest distance from the c.g., because stress is proportional to strain in the elastic range.

The rotation is assumed to produce forces of r_1, r_2, r_3, and r_4, respectively, from the bolts in the figure. The moment transferred to the bolts must be balanced by resisting moments of the bolts as shown in Equation (1)

$$M_{c.g.} = Pe = r_1 d_1 + r_2 d_2 + r_3 d_3 + r_4 d_4 \qquad (1)$$

As the force caused on each bolt is assumed to be directly proportional to the distance from the c.g., we can write

$$\frac{r_1}{d_1} = \frac{r_2}{d_2} = \frac{r_3}{d_3} = \frac{r_4}{d_4}$$

Writing each r value in terms of r_1 and d_1, we get

$$r_1 = \frac{r_1 d_1}{d_1} \quad r_2 = \frac{r_1 d_2}{d_1} \quad r_3 = \frac{r_1 d_3}{d_1} \quad r_4 = \frac{r_1 d_4}{d_1}$$

Substituting these values into the equation (original) and simplifying yields

$$M = \frac{r_1 d_1^2}{d_1} + \frac{r_1 d_2^2}{d_1} + \frac{r_1 d_3^2}{d_1} + \frac{r_1 d_4^2}{d_1} = \frac{r_1}{d_1}(d_1^2 + d_2^2 + d_3^2 + d_4^2)$$

Therefore,

$$M = \frac{r_1 \Sigma d^2}{d_1}$$

The force on each bolt can now be written as

$$r_1 = \frac{M d_1}{\Sigma d^2} \quad r_2 = \frac{d_2}{d_1} r_1 = \frac{M d_2}{\Sigma d^2} \quad r_3 = \frac{M d_3}{\Sigma d^2} \quad r_4 = \frac{M d_4}{\Sigma d^2}$$

Each value of r is perpendicular to the line drawn from the c.g. to the particular bolt. It is usually more convenient to break these reactions down into vertical and horizontal components. See Fig. 13.4.

In this figure, the horizontal and vertical components of the distance d_1 are represented by h and v, respectively, and the horizontal and vertical components of force

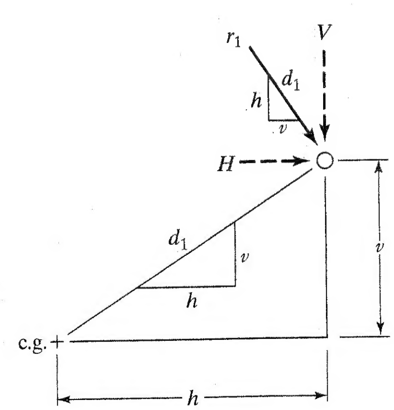


FIGURE 13.4

r_1 are represented by H and V, respectively. It is now possible to write the following ratio from which H can be obtained:

$$\frac{r_1}{d_1} = \frac{H}{v}$$

$$H = \frac{r_1 v}{d_1} = \left(\frac{Md_1}{\Sigma d^2}\right)\left(\frac{v}{d_1}\right)$$

Therefore,

$$H = \frac{Mv}{\Sigma d^2}$$

By a similar procedure,

$$V = \frac{Mh}{\Sigma d^2}$$

Example 13-1

Determine the force in the most stressed bolt of the group shown in Fig. 13.5, using the elastic analysis method.

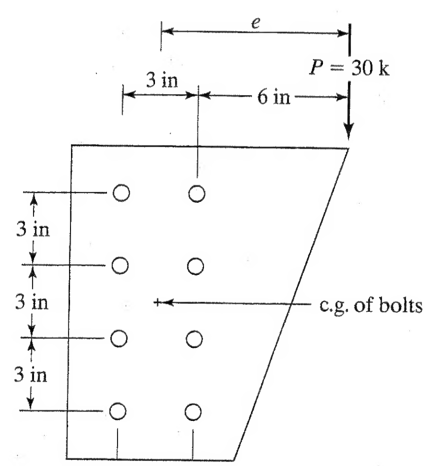


FIGURE 13.5

Solution. A sketch of each bolt and the forces applied to it by the direct load and the clockwise moment are shown in Fig. 13.6. From this sketch, the student can see that the upper right-hand bolt and the lower right-hand bolt are the most stressed and that their respective stresses are equal:

$$e = 6 + 1.5 = 7.5 \text{ in}$$

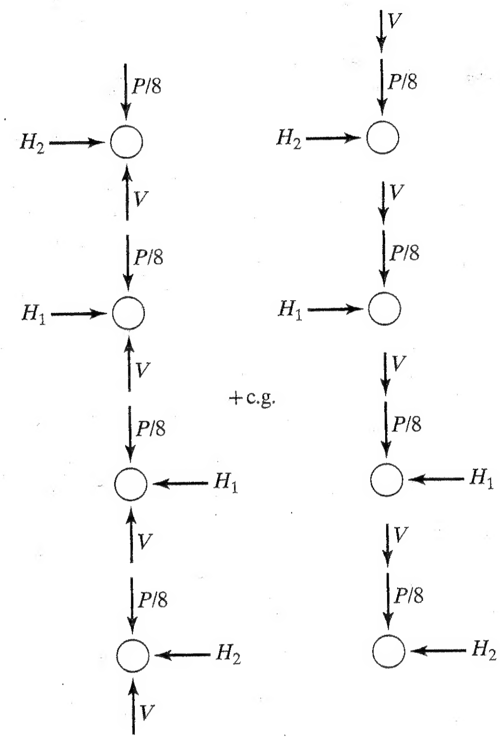


FIGURE 13.6

$$M = Pe = (30\text{ k})(7.5\text{ in}) = 225\text{ in-k}$$

$$\Sigma d^2 = \Sigma h^2 + \Sigma v^2$$

$$\Sigma d^2 = (8)(1.5)^2 + (4)(1.5^2 + 4.5^2) = 108\text{ in}^2$$

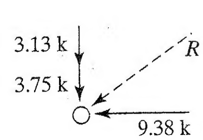
For lower right-hand bolt

$$H = \frac{Mv}{\Sigma d^2} = \frac{(225\text{ in-k})(4.5\text{ in})}{108\text{ in}^2} = 9.38\text{ k} \leftarrow$$

$$V = \frac{Mh}{\Sigma d^2} = \frac{(225\text{ in-k})(1.5\text{ in})}{108\text{ in}^2} = 3.13\text{ k} \downarrow$$

$$\frac{P}{8} = \frac{30\text{ k}}{8} = 3.75\text{ k} \downarrow$$

These components for the lower right-hand bolt are sketched as follows:



The resultant force applied to this bolt is

$$R = \sqrt{(3.13 + 3.75)^2 + (9.38)^2} = 11.63 \text{ k}$$

If the eccentric load is inclined, it can be broken down into vertical and horizontal components, and the moment of each about the c.g. of the bolt group can be determined. Various design formulas can be developed that will enable the engineer to directly design eccentric connections, but the process of assuming a certain number and arrangement of bolts, checking stresses, and redesigning probably is just as satisfactory.

The trouble with this inaccurate, but very conservative, method of analysis is that, in effect, we are assuming that there is a linear relation between loads and deformations in the fasteners; further, we assume that their yield stress is not exceeded when the ultimate load on the connection is reached. Various experiments have shown that these assumptions are incorrect.

Summing up this discussion, we can say that the elastic method is easier to apply than the instantaneous center of rotation method to be described in Section 13.1.3. However, it is probably too conservative, as it neglects the ductility of the bolts and the advantage of load redistribution.

13.1.2 Reduced Eccentricity Method

The elastic analysis method just described appreciably overestimates the moment forces applied to the connectors. As a result, quite a few proposals have been made through the years that make use of an effective eccentricity, in effect taking into account the slip resistance on the faying or contact surfaces. One set of reduced eccentricity values that were fairly common at one time follow:

1. With one gage line of fasteners and where n is the number of fasteners in the line,

$$e_{\text{effective}} = e_{\text{actual}} - \frac{1 + 2n}{4}$$

2. With two or more gage lines of fasteners symmetrically placed and where n is the number of fasteners in each line,

$$e_{\text{effective}} = e_{\text{actual}} - \frac{1 + n}{2}$$

The reduced eccentricity values for two fastener arrangements are shown in Fig. 13.7.

To analyze a particular connection with the reduced eccentricity method, the value of $e_{\text{effective}}$ is computed as described before and is used to compute the eccentric moment. Then the elastic procedure is used for the remainder of the calculations.

13.1.3 Instantaneous Center of Rotation Method

Both the elastic and reduced eccentricity methods for analyzing eccentrically loaded fastener groups are based on the assumption that the behavior of the fasteners is elastic. A much more realistic method of analysis is the instantaneous center of

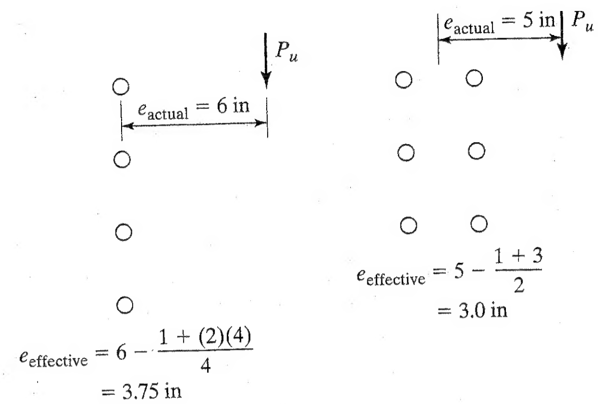


FIGURE 13.7

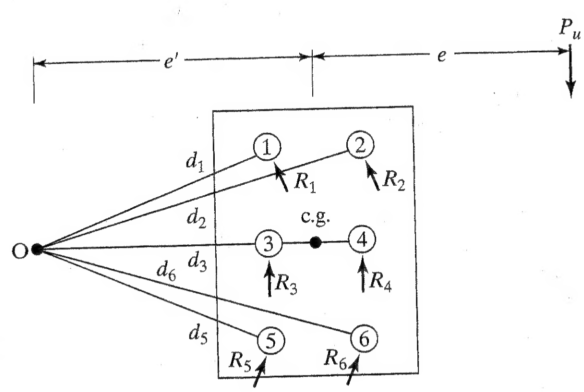


FIGURE 13.8

rotation method, which is described in the next few paragraphs. The values given in the AISC Manual for eccentrically loaded fastener groups were computed by this method.

If one of the outermost bolts in an eccentrically loaded connection begins to slip or yield, the connection will not fail. Instead, the magnitude of the eccentric load may be increased, the inner bolts will resist more load, and failure will not occur until all of the bolts slip or yield.

The eccentric load tends to cause both a relative rotation and translation of the connected material. In effect, this is equivalent to pure rotation of the connection about a single point called the *instantaneous center of rotation*. An eccentrically loaded bolted connection is shown in Fig. 13.8, and the instantaneous center is represented by point 0. It is located a distance e' from the center of gravity of the bolt group.

The deformations of these bolts are assumed to vary in proportion to their distances from the instantaneous center. The ultimate shear force that one of them can resist is not equal to the pure shear force that a bolt can resist. Rather, it is dependent

upon the load-deformation relationship in the bolt. Studies by Crawford and Kulak[4] have shown that this force may be closely estimated with the following expression:

$$R = R_{ult}(1 - e^{-10\Delta})^{0.55}$$

In this formula, R_{ult} is the ultimate shear load for a single fastener equaling 74 k for a 3/4-in diameter A325 bolt, e is the base of the natural logarithm (2.718), and Δ is the total deformation of a bolt. Its maximum value experimentally determined is 0.34 in. The Δ values for the other bolts are assumed to be in proportion to R as their d distances are to d for the bolt with the largest d. The coefficients 10.0 and 0.55 also were experimentally obtained. Figure 13.9 illustrates this load-deformation relationship.

This expression clearly shows that the ultimate shear load taken by a particular bolt in an eccentrically loaded connection is affected by its deformation. Thus, the load applied to a particular bolt is dependent upon its position in the connection with respect to the instantaneous center of rotation.

The resisting forces of the bolts of the connection of Fig. 13.8 are represented with the letters R_1, R_2, R_3, and so on. Each of these forces is assumed to act in a direction perpendicular to a line drawn from point 0 to the center of the bolt in question. For this symmetrical connection, the instantaneous center of rotation will fall somewhere on a horizontal line through the center of gravity of the bolt group. This is the case because the sum of the horizontal components of the R forces must be zero, as also must be the sum of the moments of the horizontal components about point 0. The position of point 0 on the horizontal line may be found by a tedious trial-and-error procedure to be described here.

With reference to Fig. 13.8, the moment of the eccentric load about point 0 must be equal to the summation of the moments of the R resisting forces about the same point. If we knew the location of the instantaneous center, we could compute R values for the bolts with the Crawford-Kulak formula and determine P_u from the expression to follow, in which e and e' are distances shown in Figs. 13.8 and 13.11.

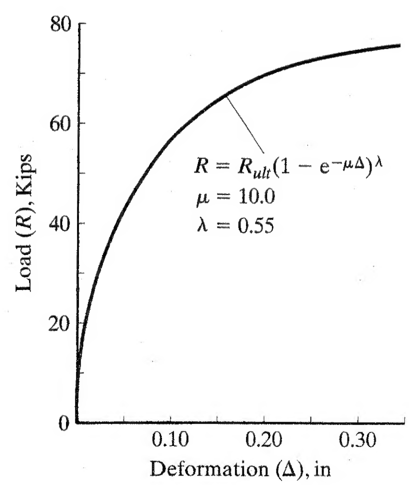


FIGURE 13.9
Ultimate shear force R in a single bolt at any given deformation.

[4]S. F. Crawford and G. L. Kulak, "Eccentrically Loaded Bolt Connections," *Journal of Structural Division*, ASCE 97, ST3 (March 1971), pp. 765–783.

$$P_u(e' + e) = \Sigma Rd$$

$$P_u = \frac{\Sigma Rd}{e' + e}$$

To determine the design strength of such a connection according to the AISC Specification, we can replace $R_{u\ell t}$ in the Crawford-Kulak formula with the design shearing strength of one bolt in a connection where the load is not eccentric. For instance, if we have 7/8-in A325 bolts (threads excluded from shear plane) in single shear bearing on a sufficient thickness so that bearing does not control, $R_{u\ell t}$ will equal, for the LRFD method,

$$R_{ult} = \phi F_n A_b = (0.75)(68 \text{ ksi})(0.60 \text{ in}^2) = 30.6 \text{ k}$$

The location of the instantaneous center is not known, however. Its position is estimated, the R values determined, and P_u calculated as described. It will be noted that P_u must be equal to the summation of the vertical components of the R resisting forces (ΣR_v). If the value is computed and equals the P_u computed by the preceding formula, we have the correct location for the instantaneous center. If not, we try another location, and so on.

In Example 13-2, the author demonstrates the very tedious trial-and-error calculations necessary to locate the instantaneous center of rotation for a symmetrical connection consisting of four bolts. In addition, the LRFD design strength of the connection ϕR_n and the allowable strength R_n/Ω are determined.

To solve such a problem, it is very convenient to set the calculations up in a table similar to the one used in the solution to follow. In the table shown, the h and v values given are the horizontal and vertical components of the d distances from point 0 to the centers of gravity of the individual bolts. The bolt that is located at the greatest distance from point 0 is assumed to have a Δ value of 0.34 in. The Δ values for the other bolts are assumed to be proportional to their distances from point 0. The Δ values so determined are used in the R formula.

A set of tables entitled "Coefficients C for Eccentrically Loaded Bolt Groups" is presented in Tables 7-7 to 7-14 of the AISC Manual. The values in these tables were determined by the procedure described here. A large percentage of the practical cases that the designer will encounter are included in the tables. Should some other situation not covered be faced, the designer may very well decide to use the more conservative elastic procedure previously described.

Example 13-2

The bearing-type 7/8-in A325 bolts of the connection of Fig. 13.10 have a nominal shear strength $r_n = (0.60 \text{ in}^2)(68 \text{ ksi}) = 40.8$ k. Locate the instantaneous center of rotation of the connection, using the trial-and-error procedure, and determine the value of P_u.

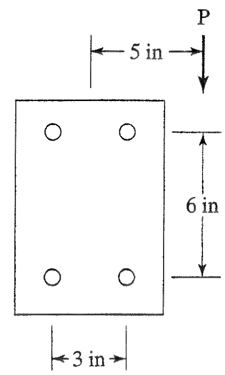


FIGURE 13.10

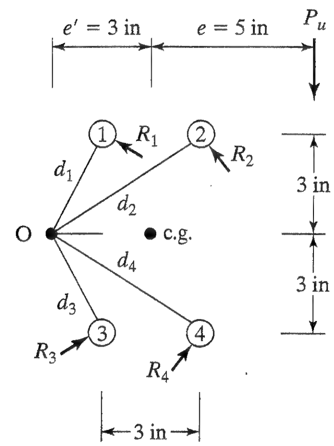


FIGURE 13.11

Solution. *By trial and error:* Try a value of $e' = 3$ in, reference being made to Fig. 13.11. In the accompanying table, Δ for bolt 1 equals (3.3541/5.4083) (0.34) $= 0.211$ in and R for the same bolt equals $30.6(1 - e^{-(10)(0.211)})^{0.55}$.

Bolt No.	h (in)	v (in)	d (in)	Δ (in)	R(kips)	R_v(kips)	Rd (k-in)
1	1.5	3	3.3541	0.211	28.50	12.74	95.58
2	4.5	3	5.4083	0.34	30.03	24.99	162.43
3	1.5	3	3.3541	0.211	28.50	12.74	95.58
4	4.5	3	5.4083	0.34	30.03	24.99	162.43
						$\Sigma = 75.46$	$\Sigma = 516.03$

$$P_u = \frac{\Sigma Rd}{e' + e} = \frac{516.03}{3 + 5} = 64.50 \text{ k not} = 75.46 \text{ k} \qquad \text{N.G.}$$

After several trials, assume that $e' = 2.40$ in.

Bolt No.	h (in)	v (in)	d (in)	Δ (in)	R (kips)	R_v (kips)	Rd (k-in)
1	0.90	3	3.1321	0.216	28.61	8.22	89.62
2	3.90	3	4.9204	0.34	30.03	23.81	147.78
3	0.90	3	3.1321	0.216	28.61	8.22	89.62
4	3.90	3	4.9204	0.34	30.03	23.81	147.78
						$\Sigma = 64.06$	$\Sigma = 474.80$

Then, we have

$$P_u = \frac{\Sigma Rd}{e' + e} = \frac{474.80}{2.4 + 5} = 64.16 \text{ k almost} = 64.06 \text{ k} \qquad \text{OK}$$

$$P_u = 64.1 \text{ k}$$

Although the development of this method of analysis was actually based on bearing-type connections where slip may occur, both theory and load tests have shown that the method may conservatively be applied to slip-critical connections.[5]

The instantaneous center of rotation may be expanded to include inclined loads and unsymmetrical bolt arrangements, but the trial-and-error calculations with a hand calculator are extraordinarily long for such situations.

Examples 13-3 and 13-4 provide illustrations of the use of the ultimate strength tables in Part 7 of the AISC Manual, for both analysis and design.

Example 13-3

Repeat Example 13-2, using the tables in Part 7 of the Manual. These tables are entitled "Coefficients C for Eccentrically Loaded Bolt Groups." Determine both LRFD design strength and ASD allowable strength of connection.

Solution. Enter Manual Table 7-8 with angle = 0°, $s = 6$ in, $e_x = 5$ in, and $n = 2$ vertical rows.

$$C = 2.24$$

$$r_n = F_{nv} A_g = (68 \text{ ksi})(0.6 \text{ in}^2) = 40.8 \text{ k}$$

(From statement in Example 13-2, shear controls are not checked, and thus bearing is not checked.)

$$R_n = Cr_n = (2.24)(40.8) = 91.4 \text{ k}$$

[5]G. L. Kulak, "Eccentrically Loaded Slip-Resistant Connections," *Engineering Journal*, AISC, vol. 12, no. 2 (2nd Quarter, 1975), pp. 52–55.

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi R_n = (0.75)(91.4) = \mathbf{68.6\ k}$ Generally agrees with trial-and-error solution in preceding example.	$\frac{R_n}{\Omega} = \frac{91.4}{2.00} = \mathbf{45.7\ k}$

Example 13-4

Using both LRFD and ASD, determine the number of 7/8-in A325 bolts in standard-size holes required for the connection shown in Fig. 13.12. Use A36 steel and assume that the connection is to be a bearing type with threads excluded from the shear plane. Further assume that the bolts are in single shear and bearing on 1/2 in. Use the instantaneous center of rotation method as presented in the tables of Part 7 of the AISC Manual. Assume that $L_c = 1.0$ in and deformation at bolt holes at service loads is not a design consideration.

Solution

$$e_x = e = 5\tfrac{1}{2}\text{ in} = 5.5\text{ in}$$

Bolts in single shear and bearing on 1/2 in:

$$r_n = \text{nominal shear strength per fastener}$$
$$= F_{nv}A_b = (68\text{ ksi})(0.6\text{ in}^2) = 40.8\text{ k} \leftarrow$$
$$r_n = \text{nominal bearing strength per fastener}$$
$$= 1.5 l_c t F_u = (1.5)(1.0\text{ in})\left(\frac{1}{2}\text{ in}\right)(58\text{ ksi}) = 43.5\text{ k}$$
$$< 3.0\, dtF_u = (3.0)\left(\frac{7}{8}\right)\left(\frac{1}{2}\right)(58) = 76.1\text{ k}$$

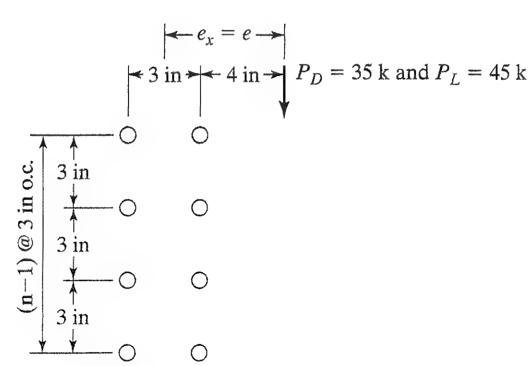


FIGURE 13.12

With reference to Table 7-8 in the Manual, the value of C_{min} required to provide a sufficient number of bolts can be determined as follows:

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$P_u = (1.2)(35) + (1.6)(45) = 114\text{ k}$	$P_a = 35 + 45 = 80\text{ k}$
$C_{min} = \dfrac{P_u}{\phi r_n} = \dfrac{114}{(0.75)(40.8)} = 3.73$	$C_{min} = \dfrac{\Omega P_a}{r_n} = \dfrac{(2.00)80}{40.8} = 3.92$
* Use four $\frac{7}{8}$ A325 bolts in each row, as described next.	

* With $e_x = 5\ 1/2$ in and a vertical spacing s of 3 in, we move horizontally in the table until we find the number of bolts in each vertical row so as to provide a C of 3.73 or more. With $e_x = 5$ in and $n = 4$, we find $C = 4.51$. Then, with $e_x = 6$ in and $n = 4$, we find $C = 4.03$. Interpolating for $e_x = 5\ 1/2$ in, we find $C = 4.27 > 3.73$, OK for LRFD.

Since $C = 4.27 > 3.92$, OK for ASD

Use four 7/8 in A325 × bolts in each row (for both LRFD and ASD).

Note: If the situation faced by the designer does not fit the eccentrically loaded bolt group tables given in Part 7 of the AISC Manual, it is recommended that the conservative elastic procedure be used to handle the problem, whether analysis or design.

13.2 BOLTS SUBJECTED TO SHEAR AND TENSION (BEARING-TYPE CONNECTIONS)

The bolts used for a large number of structural steel connections are subjected to a combination of shear and tension. One quite obvious case is shown in Fig. 13.13, where a diagonal brace is attached to a column. The vertical component of force in the figure, V, is trying to shear the bolts off at the face of the column, while the horizontal component of force, H, is trying to fracture them in tension.

Tests on bearing-type bolts subject to combined shear and tension show that their strengths can be represented with an elliptical interaction curve, as shown in Fig. 13.14. The three straight dashed lines shown in the figure can be used quite accurately to represent the elliptical curve. In this figure, the horizontal dashed line represents the design tensile stress of LRFD ϕF_{nt} or the allowable tensile stress of ASD F_{nt}/Ω if no shear forces is applied to the bolts. The vertical dashed line represents the design shear stress of LRFD ϕF_{nv} or the allowable shear stress of the ASD F_{nv}/Ω if no tensile forces are applied to the bolts.

The sloped straight line in the figure is represented by the expression for F'_{nt}, the nominal tensile stress modified to include the effects of shearing force. Expressions for F'_{nt} follow. These values are provided in Section J3.7 of the AISC Specification.

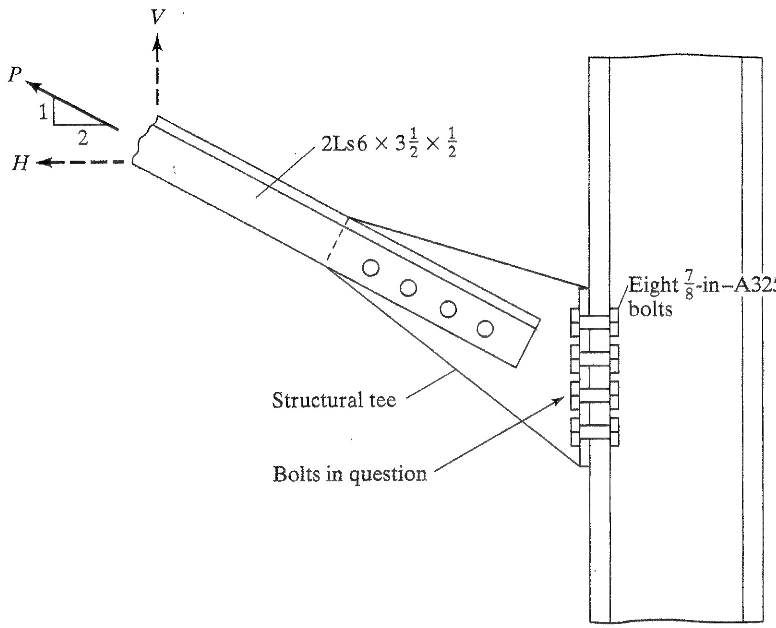


FIGURE 13.13
Combined shear and tension connection.

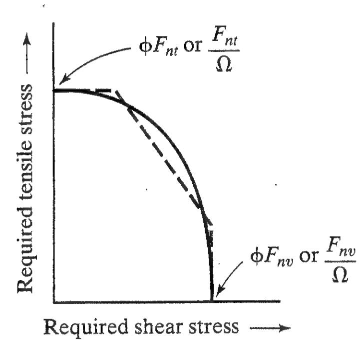


FIGURE 13.14
Bolts in a bearing-type connection subject to combined shear and tension.

For LRFD ($\phi = 0.75$)

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \qquad \text{(AISC Equation J3-3a)}$$

For ASD ($\Omega = 2.00$)

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt} \qquad \text{(AISC Equation J3-3b)}$$

APD Building, Dublin, GA. (Courtesy Britt, Peters and Associates.)

in which

F_{nt} is the nominal tensile stress from Table 12-5 (AISC Table J3.2), ksi.

F_{nv} is the nominal shear stress from Table 12-5 (AISC Table J3.2), ksi.

f_{rv} is the required shear stress using LRFD or ASD load combinations, ksi.

The available shear stress of the fastener shall equal or exceed the required shear stress, f_{rv}.

The AISC Specification (J3.7) says that if the required stress, f, in either shear or tension, is equal to or less than 30 percent of the corresponding available stress, it is not necessary to investigate the effect of combined stress.

Example 13-5

The tension member previously shown in Fig. 13.13 has eight 7/8-in A325 high-strength bolts in a bearing-type connection. Is this a sufficient number of bolts to resist the applied

loads $P_D = 80$ k and $P_L = 100$ k, using the LRFD and ASD specifications, if the bolt threads are excluded from the shear planes?

Solution

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$P_u = (1.2)(80) + (1.6)(100) = 256$ k	$P_a = 80 + 100 = 180$ k
$V = \dfrac{1}{\sqrt{5}}(256) = 114.5$ k	$V = \dfrac{1}{\sqrt{5}}(180) = 80.5$ k
$H = \dfrac{2}{\sqrt{5}}(256) = 229$ k	$H = \dfrac{2}{\sqrt{5}}(180) = 161$ k
$F_{nt} = 90$ ksi	$F_{nt} = 90$ ksi
$F_{nv} = 68$ ksi	$F_{nv} = 68$ ksi
$f_{rv} = \dfrac{114.5\text{ k}}{(8)(0.6\text{ in}^2)} = 23.85$ ksi	$f_v = \dfrac{80.5\text{ k}}{(8)(0.6\text{ in}^2)} = 16.77$ ksi
$f_{rt} = \dfrac{229\text{ k}}{(8)(0.6\text{ in}^2)} = 47.7$ ksi	$f_t = \dfrac{161\text{ k}}{(8)(0.6\text{ in}^2)} = 33.54$ ksi
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}$	$F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt}$
$= (1.3)(90) - \dfrac{90}{(0.75)(68)}(23.85)$	$= (1.3)(90) - \dfrac{(2.00)(90)}{68}(16.77)$
$= 74.9$ ksi < 90 ksi	$= 72.6$ ksi < 90 ksi
$\phi F'_{nt} = (0.75)(74.9) = 56.2$ ksi > 47.7 ksi	$\dfrac{F'_{nt}}{\Omega} = \dfrac{72.6}{2.00} = 36.3$ ksi > 33.54 ksi
Connection is OK.	**Connection is OK.**

13.3 BOLTS SUBJECTED TO SHEAR AND TENSION (SLIP-CRITICAL CONNECTIONS)

When an axial tension force is applied to a slip-critical connection, the clamping force will be reduced, and the design shear strength must be decreased in some proportion to the loss in clamping or prestress. This is accomplished in the AISC Specification (Section J3.9) by multiplying the available slip resistance of the bolts (as determined in AISC Section J.8) by a factor k_{sc}.

For LRFD

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \qquad \text{(AISC Equation J3-5a)}$$

For ASD

$$k_{sc} = 1 - \frac{1.5T_a}{D_u T_b n_b} \qquad \text{(AISC Equation J3-5b)}$$

Here, the factors are defined as follows:

T_u = the tension force due to the LRFD load combination (that is, $\dfrac{P_u}{n_b}$)

D_u = a multiplier = 1.13, previously defined in Section 12.14 (AISC Section J3.8)

T_b = minimum fastener tension, as given in Table 12.1 (Table J3.1, AISC)

n_b = the number of bolts carrying the applied tension

T_a = the tension force due to the ASD load combination (that is, $\frac{P_a}{n_b}$)

Example 13-6

A group of twelve 7/8-in A325 high-strength bolts with standard holes is used in a lap joint for a slip-critical joint designed to prevent slip. The connection is to resist the service shear loads $V_D = 40$ k and $V_L = 50$ k, as well as the tensile service loads $T_D = 50$ k and $T_L = 50$ k. Is the connection satisfactory if the faying surface is Class B and the factor for fillers, h_f, is 1.00?

Solution

R_n for 1 bolt in an ordinary slip-critical connection

$$R_n = \mu D_u h_f T_b n_s = (0.50)(1.13)(1.00)(39)(1) = 22.03 \text{ k/bolt}$$

LRFD $\phi = 1.00$	ASD $\Omega = 1.50$
$V_u = (1.2)(40) + (1.6)(50) = 128$ k	$V_a = 40 + 50 = 90$ k
$T_u = (1.2)(50) + (1.6)(50) = 140$ k	$T_a = 50 + 50 = 100$ k
$\phi R_n = (1.0)(22.03) = 22.03$ k/bolt	$\frac{R_n}{\Omega} = \frac{22.03}{1.50} = 14.69$ k/bolt
Reduction due to tensile load	Reduction due to tensile load
$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$	$k_{sc} = 1 - \frac{1.5T_a}{D_u T_b n_b}$
$= 1 - \frac{140}{(1.13)(39)(12)} = 0.735$	$= 1 - \frac{(1.5)(100)}{(1.13)(39)(12)} = 0.716$
Reduced ϕR_n/bolt	Reduced $\frac{R_n}{\Omega}$/bolt
$= (0.735)(22.03 \text{ k}) = 16.20$ k/bolt	$= (0.716)(14.69) = 10.52$ k/bolt
Design slip resistance for 12 bolts $= (12)(16.20) = 194.4$ k	Allowable slip resistance for 12 bolts $= (12)(10.52) = 126.2$ k
>128 k **OK**	>90 k **OK**
Connection is satisfactory.	**Connection is satisfactory.**

13.4 TENSION LOADS ON BOLTED JOINTS

Bolted and riveted connections subjected to pure tensile loads have been avoided as much as possible in the past by designers. The use of tensile connections was probably used more often for wind-bracing systems in tall buildings than for any other situation.

Other locations exist, however, where they have been used, such as hanger connections for bridges, flange connection for piping systems, etc. Figure 13.15 shows a hanger-type connection with an applied tensile load.

Hot-driven rivets and fully tensioned high-strength bolts are not free to shorten, with the result that large tensile forces are produced in them during their installation. These initial tensions are actually close to their yield points. There has always been considerable reluctance among designers to apply tensile loads to connectors of this type for fear that the external loads might easily increase their already present tensile stresses and cause them to fail. The truth of the matter, however, is that when external tensile loads are applied to connections of this type, the connectors probably will experience little, if any, change in stress.

Fully tensioned high-strength bolts actually prestress the joints in which they are used against tensile loads. (Think of a prestressed concrete beam that has external compressive loads applied at each end.) The tensile stresses in the connectors squeeze together the members being connected. If a tensile load is applied to this connection at the contact surface, it cannot exert any additional load on the bolts until the members are pulled apart and additional strains put on the bolts. The members cannot be pulled apart until a load is applied that is larger than the total tension in the connectors of the connection. This statement means that the joint is prestressed against tensile forces by the amount of stress initially put in the shanks of the connectors.

Another way of saying this is that if a tensile load P is applied at the contact surface, it tends to reduce the thickness of the plates somewhat, but, at the same time, the contact pressure between the plates will be correspondingly reduced, and the plates will tend to expand by the same amount. The theoretical result, then, is no change in plate thickness and no change in connector tension. This situation continues until P equals the connector tension. At this time an increase in P will result in separation of the plates, and thereafter the tension in the connector will equal P.

Should the load be applied to the outer surfaces, there will be some immediate strain increase in the connector. This increase will be accompanied by an expansion of the plates, even though the load does not exceed the prestress, but the increase will be very slight because the load will go to the plate and connectors roughly in proportion

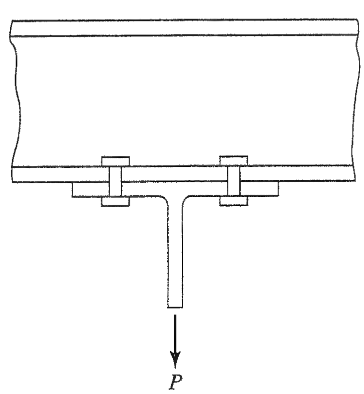


FIGURE 13.15
Hanger connection.

to their stiffness. As the plate is stiffer, it will receive most of the load. An expression can be developed for the elongation of the bolt, on the basis of the bolt area and the assumed contact area between the plates. Depending on the contact area assumed, it will be found that, unless P is greater than the bolt tension, its stress increase will be in the range of 10 percent. Should the load exceed the prestress, the bolt stress will rise appreciably.

The preceding rather lengthy discussion is approximate, but should explain why an ordinary tensile load applied to a bolted joint will not change the stress situation very much.

The AISC nominal tensile strength of bolted or threaded parts is given by the expression to follow, which is independent of any initial tightening force:

$$R_n = F_n A_b, \text{ with } F_n = F_{nt} \text{ for tension or } F_{nv} \text{ for shear.} \quad \text{(AISC Equation J3-1)}$$

When fasteners are loaded in tension, there is usually some bending due to the deformation of the connected parts. As a result, the value of ϕ for LRFD is a rather small 0.75, and Ω for ASD is a rather large 2.00. Table 12.5 of this text (AISC Table J3.2) gives values of F_{nt}, the nominal tensile strength (ksi) for the different kinds of connectors, with the values of threaded parts being quite conservative.

In this expression, A_b is the nominal body area of the unthreaded portion of a bolt, or its threaded part not including upset rods. An upset rod has its ends made larger than the regular rod, and the threads are placed in the enlarged section so that the area at the root of the thread is larger than that of the regular rod. An upset rod was shown in Fig. 4.3. The use of upset rods is not usually economical and should be avoided unless a large order is being made.

If an upset rod is used, the nominal tensile strength of the threaded portion is set equal to $0.75F_u$ times the cross-sectional area at its major thread diameter. This value must be larger than F_y times the nominal body area of the rod before upsetting.

Example 13-7 illustrates the determination of the strength of a tension connection.

Example 13-7

Determine the design tensile strength (LRFD) and the allowable tensile strength (ASD) of the bolts for the hanger connection of Fig. 13.15 if eight 7/8-in A490 high-strength bolts with threads excluded from the shear plane are used. Neglect prying action.

Solution

$$R_n \text{ for 8 bolts} = 8F_{nt}A_b = (8)(113 \text{ ksi})(0.6 \text{ in}^2) = 542.4 \text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi R_n = (0.75)(542.4) = \mathbf{406.8\ k}$	$\dfrac{R_n}{\Omega} = \dfrac{542.4}{2.00} = \mathbf{271.2\ k}$

13.5 PRYING ACTION

A further consideration that should be given to tensile connections is the possibility of prying action. A tensile connection is shown in Fig. 13.16(a) that is subjected to prying action as illustrated in part (b) of the same figure. Should the flanges of the connection be quite thick and stiff or have stiffener plates like those in Fig. 13.16(c), the prying action will probably be negligible, but this would not be the case if the flanges are thin and flexible and have no stiffeners.

It is usually desirable to limit the number of rows of bolts in a tensile connection, because a large percentage of the load is carried by the inner rows of multi-row connections, even at ultimate load. The tensile connection shown in Fig. 13.17 illustrates this point, as the prying action will throw a large part of the load to the inner connectors, particularly if the plates are thin and flexible. For connections subjected to pure tensile loads, estimates should be made of possible prying action and its magnitude.

The additional force in the bolts resulting from prying action should be added to the tensile force resulting directly from the applied forces. The actual determination

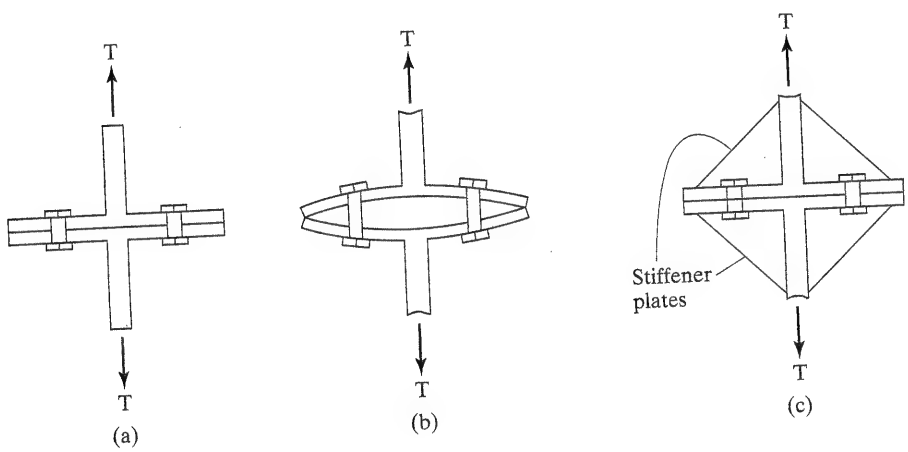


FIGURE 13.16

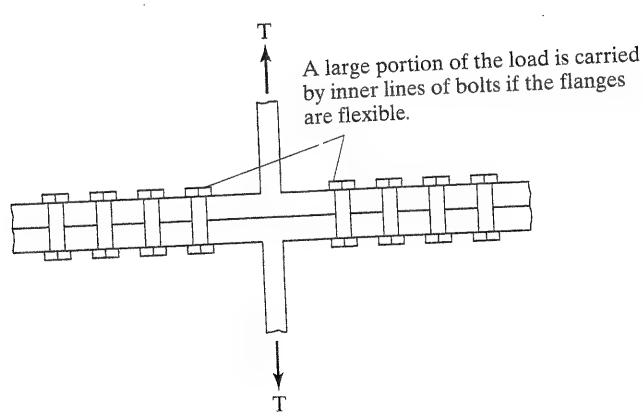


FIGURE 13.17

of prying forces is quite complex, and research on the subject is still being conducted. Several empirical formulas have been developed that approximate test results. Among these are the AISC expressions included in this section.

Hanger and other tension connections should be so designed as to prevent significant deformations. The most important item in such designs is the use of rigid flanges. Rigidity is more important than bending resistance. To achieve this goal, the distance b shown in Fig. 13.18 should be made as small as possible, with a minimum value equal to the space required to use a wrench for tightening the bolts. Information concerning wrench clearance dimensions is presented in a table entitled "Entering and Tightening Clearance" in Tables 7-16 and 7-17 of Part 7 of the AISC Manual.

Prying action, which is present only in bolted connections, is caused by the deformation of the connecting elements when tensile forces are applied. The results are increased forces in some of the bolts above the forces caused directly by the tensile forces. Should the thicknesses of the connected parts be as large as or larger than the values given by the AISC formulas to follow, which are given on pages 9–10 in the Manual, prying action is considered to be negligible. Reference is here made to Fig. 13.18 for the terms involved in the formulas.

For LRFD **For ASD**

$$t_{min} = \sqrt{\frac{4.44Tb'}{pF_u}} \qquad t_{min} = \sqrt{\frac{6.66Tb'}{pF_u}}$$

The following terms are defined for these formulas:

$$T = \text{required strength of each bolt} = r_{ut}$$

$$\text{or} \quad r_{at} = \frac{T_u \text{ or } T_a}{\text{no. of bolts}}, \text{ kips}$$

$$b' = \left(b - \frac{d_b}{2}\right), \text{ in}$$

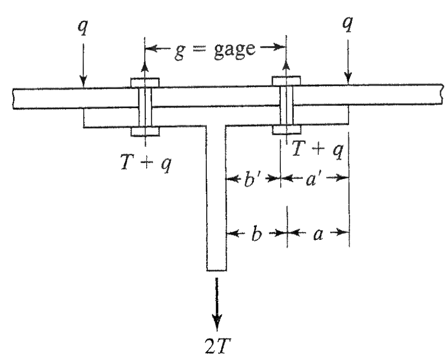


FIGURE 13.18

b = distance from ℄ of bolt to face of tee (for an angle b is measured to ℄ of angle leg) in

d_b = bolt diameter

p = tributary length per pair of bolts ($\perp$ to plane of paper) preferably not $>g$, in

F_u = specified minimum tensile strength of the connecting element, ksi

Example 13-8, which follows, presents the calculation of the minimum thickness needed for a structural tee flange so that prying action does not have to be considered for the bolts.

Example 13-8

A 10-in long WT8 × 22.5 (t_f = 0.565 in, t_w = 0.345 in, and b_f = 7.04 in) is connected to a W36 × 150 as shown in Fig. 13.19, with six 7/8-in A325 high-strength bolts spaced 3 in o.c. If A36 steel is used, F_u = 58 ksi, is the flange sufficiently thick if prying action is considered? P_D = 30 k and P_L = 40 k.

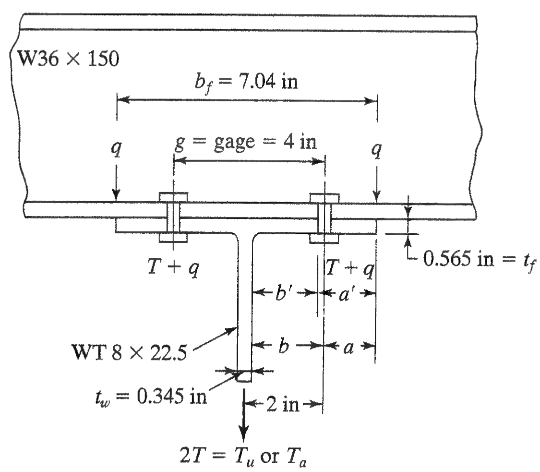


FIGURE 13.19

Solution

LRFD	ASD
$T_u = (1.2)(30) + (1.6)(40) = 100$ k	$T_a = 30 + 40 = 70$ k
$T = r_{ut} = \dfrac{100}{6} = 16.67$ k each bolt	$T = r_{at} = \dfrac{70}{6} = 11.67$ k each bolt

$$b' = \left(2 - \frac{0.345}{2}\right) - \frac{0.875}{2} = 1.39 \text{ in}$$

$$d_b = 0.875 \text{ in}$$

$$p = 3 \text{ in}$$

LRFD	ASD
$t_{min} = \sqrt{\dfrac{(4.44)(16.67)(1.39)}{(3)(58)}}$	$t_{min} = \sqrt{\dfrac{(6.66)(11.67)(1.39)}{(3)(58)}}$
$= 0.769 \text{ in} > t_f = 0.565 \text{ in}$	$= 0.788 \text{ in} > t_f = 0.565 \text{ in}$
$\therefore$ Prying action must be considered.	$\therefore$ Prying action must be considered.

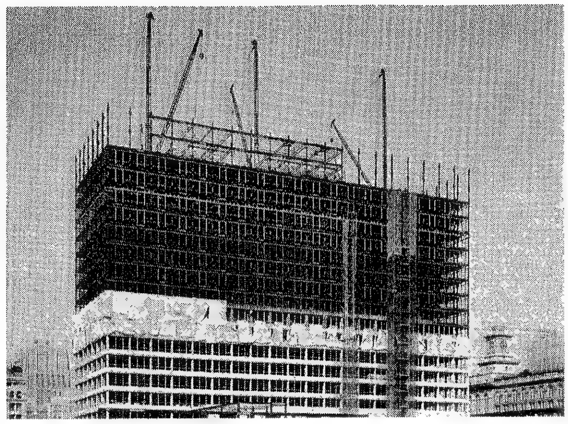
Note: Though not presented here in, pages 9-10 through 9-13 in the AISC Manual provide equations for computing the extra tensile force (q) caused by prying action.

13.6 HISTORICAL NOTES ON RIVETS

Rivets were the accepted method for connecting the members of steel structures for many years. Today, however, they no longer provide the most economical connections and are obsolete. It is doubtful that you could find a steel fabricator who can do riveting. It is, however, desirable for the designer to be familiar with rivets, even though he or she will probably never design riveted structures. He or she may have to analyze an existing riveted structure for new loads or for an expansion of the structure. The purpose of these sections is to present only a very brief introduction to the analysis and design of rivets. One advantage of studying these obsolete connectors is that, while doing so, you automatically learn how to analyze A307 common bolts. These bolts are handled exactly as are rivets, except that the design stresses are slightly different. Example 13-11 illustrates the design of a connection with A307 bolts.

The rivets used in construction work were usually made of a soft grade of steel that would not become brittle when heated and hammered with a riveting gun to form the head. The typical rivet consisted of a cylindrical shank of steel with a rounded head on one end. It was heated in the field to a cherry-red color (approximately 1800°F), inserted in the hole, and a head formed on the far end, probably with a portable rivet gun powered by compressed air. The rivet gun, which had a depression in its head to give the rivet head the desired shape, applied a rapid succession of blows to the rivet.

For riveting done in the shop, the rivets were probably heated to a light cherry-red color and driven with a pressure-type riveter. This type of riveter, usually called a "bull" riveter, squeezed the rivet with a pressure of perhaps as high as 50 to 80 tons (445 to 712 kN) and drove the rivet with one stroke. Because of this great pressure, the rivet in its soft state was forced to fill the hole very satisfactorily. This type of riveting



The U.S. Customs Court, Federal Office Building under construction in New York City. (Courtesy of Bethlehem Steel Corporation.)

was much to be preferred over that done with the pneumatic hammer, but no greater nominal strengths were allowed by riveting specifications. The bull riveters were built for much faster operation than were the portable hand riveters, but the latter riveters were needed for places that were not easily accessible (i.e., field erection).

As the rivet cooled, it shrank, or contracted, and squeezed together the parts being connected. The squeezing effect actually caused considerable transfer of stress between the parts being connected to take place by friction. The amount of friction was not dependable, however, and the specifications did not permit its inclusion in the strength of a connection. Rivets shrink diametrically as well as lengthwise and actually become somewhat smaller than the holes that they are assumed to fill. (Permissible strengths for rivets were given in terms of the nominal cross-sectional areas of the rivets before driving.)

Some shop rivets were driven cold with tremendous pressures. Obviously, the cold-driving process worked better for the smaller-size rivets (probably 3/4 in in diameter or less), although larger ones were successfully used. Cold-driven rivets fill the holes better, eliminate the cost of heating, and are stronger because the steel is cold worked. There is, however, a reduction of clamping force, since the rivets do not shrink after having been driven.

13.7 TYPES OF RIVETS

The sizes of rivets used in ordinary construction work were 3/4 in and 7/8 in in diameter, but they could be obtained in standard sizes from 1/2 in to 1 1/2 in in 1/8-in increments. (The smaller sizes were used for small roof trusses, signs, small towers, etc., while the larger sizes were used for very large bridges or towers and very tall buildings.) The use of more than one or two sizes of rivets or bolts on a single job is usually undesirable, because it is expensive and inconvenient to punch different-size holes in a member in

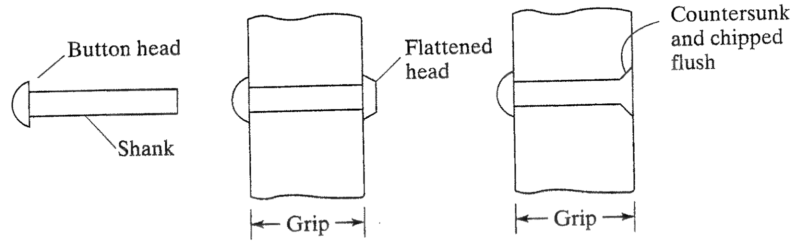


FIGURE 13.20

Types of rivets.

the shop, and the installation of different-size rivets or bolts in the field may be confusing. Some cases arise where it is absolutely necessary to have different sizes, as where smaller rivets or bolts are needed for keeping the proper edge distance in certain sections, but these situations should be avoided if possible.

Rivet heads, usually round in shape, were called *button heads*; but if clearance requirements dictated, the head was flattened or even countersunk and chipped flush. These situations are shown in Fig. 13.20.

The countersunk and chipped-flush rivets did not have sufficient bearing areas to develop full strength, and the designer usually discounted their computed strengths by 50 percent. A rivet with a flattened head was preferred over a countersunk rivet, but if a smooth surface was required, the countersunk and chipped-flush rivet was necessary. This latter type of rivet was appreciably more expensive than the button head type, in addition to being weaker; and it was not used unless absolutely necessary.

There were three ASTM classifications for rivets for structural steel applications, as described in the paragraphs that follow.

13.7.1 ASTM Specification A502, Grade 1

These rivets were used for most structural work. They had a low carbon content of about 0.80 percent, were weaker than the ordinary structural carbon steel, and had a higher ductility. The fact that these rivets were easier to drive than the higher-strength rivets was the main reason that, when rivets were used, they probably were A502, Grade 1, regardless of the strength of the steel used in the structural members.

13.7.2 ASTM Specification A502, Grade 2

These carbon-manganese rivets had higher strengths than the Grade 1 rivets and were developed for the higher-strength steels. Their higher strength permitted the designer to use fewer rivets in a connection and thus smaller gusset plates.

13.7.3 ASTM Specification A502, Grade 3

These rivets had the same nominal strengths as the Grade 2 rivets, but they had much higher resistance to atmospheric corrosion, equal to approximately four times that of carbon steel without copper.

13.8 STRENGTH OF RIVETED CONNECTIONS—RIVETS IN SHEAR AND BEARING

The factors determining the strength of a rivet are its grade, its diameter, and the thickness and arrangement of the pieces being connected. The actual distribution of stress around a rivet hole is difficult to determine, if it can be determined at all; and to simplify the calculations, it is assumed to vary uniformly over a rectangular area equal to the diameter of the rivet times the thickness of the plate.

The strength of a rivet in single shear is the nominal shearing strength times the cross-sectional area of the shank of the rivet. Should a rivet be in double shear, its shearing strength is considered to be twice its single-shear value.

AISC Appendix 5.2.6 indicates that, in checking older structures with rivets, the designer is to assume that the rivets are ASTM A502, grade 1, unless a higher grade is determined by documentation or testing. The nominal shearing strength of A502, grade 1 rivets was 25 ksi, and ϕ was 0.75.

Examples 13-9 and 13-10 illustrate the calculations necessary either to determine the LRFD design and the ASD allowable strengths of existing connections or to design riveted connections. Little comment is made here concerning A307 bolts. The reason is that all the calculations for these fasteners are made exactly as they are for rivets, except that the shearing strengths given by the AISC Specification are different. Only one brief example with common bolts (Example 13-11) is included.

The AISC Specification does not today include rivets, and thus ϕ and Ω values are not included therein. For the example problems to follow, the author uses the rivet ϕ values which were given in the third edition of the LRFD Specification. The Ω values were then determined by the author with the expression $\Omega = 1.50/\phi$, as they were throughout the present specification.

Example 13-9

Determine the LRFD design strength ϕP_n and the ASD allowable strength P_n/Ω of the bearing-type connection shown in Fig. 13.21. A36 steel and A502, Grade 1 rivets

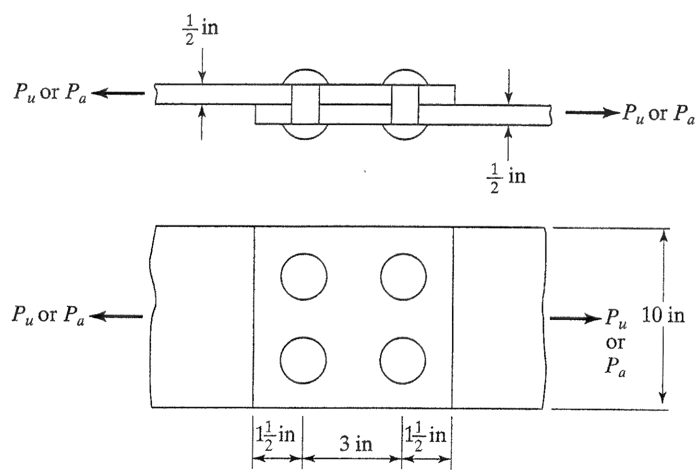


FIGURE 13.21

are used in the connection, and it is assumed that standard-size holes are used and that edge distances and center-to-center distances are = 1.5 in and 3 in, respectively. Neglect block shear. The rivets are 3/4 in in diameter and their F_{nv} is 25 ksi.

Solution. Design tensile force applied to plates

$$A_g = \left(\frac{1}{2}\text{ in}\right)(10\text{ in}) = 5.00\text{ in}^2$$

$$A_n = \left[\left(\frac{1}{2}\text{ in}\right)(10\text{ in}) - (2)\left(\frac{7}{8}\text{ in}\right)\left(\frac{1}{2}\text{ in}\right)\right] = 4.125\text{ in}^2$$

For tensile yielding

$$P_n = F_y A_g = (36\text{ ksi})(5.00\text{ in}^2) = 180\text{ k}$$

LRFD $\phi_t = 0.90$	ASD $\Omega_t = 1.67$
$\phi_t P_n = (0.90)(180) = 162\text{ k}$	$\dfrac{P_n}{\Omega_t} = \dfrac{180}{1.67} = 107.8\text{ k}$

For tensile rupture

$$A_e = UA_n = 1.0 \times 4.125\text{ in}^2 = 4.125\text{ in}^2$$
$$P_n = F_u A_e = (58\text{ ksi})(4.125\text{ in}^2) = 239.25\text{ k}$$

LRFD $\phi_t = 0.75$	ASD $\Omega_t = 2.00$
$\phi_t P_n = (0.75)(239.25) = 179.4\text{ k}$	$\dfrac{P_n}{\Omega_t} = \dfrac{239.25}{2.00} = 119.6\text{ k}$

Rivets in single shear and bearing on 1/2 in

$$R_n = (A_{rivet})(F_{nv})(\text{no. of rivets}) = (0.44\text{ in}^2)(25\text{ ksi})(4) = 44.0\text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$	
$\phi R_n = (0.75)(44.0) = 33.0\text{ k}$	$\dfrac{R_n}{\Omega} = \dfrac{44.0}{2.00} = 22.0\text{ k}$	← Controls

For bearing with $l_c = 1.50 - \dfrac{\frac{3}{4} + \frac{1}{8}}{2} = 1.06\text{ in}$

$$R_n = 1.2 l_c t F_u\,(\text{no. of rivets}) = (1.2)(1.06)\left(\frac{1}{2}\right)(58)(4) = 147.55\text{ k}$$

$$< 2.4\, dtF_u\,(\text{no. of rivets}) = (2.4)\left(\frac{3}{4}\right)\left(\frac{1}{2}\right)(58)(4) = 208.8\text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi R_n = (0.75)(147.55) = 110.7\text{ k}$	$\dfrac{R_n}{\Omega} = \dfrac{147.55}{2.0} = 73.8\text{ k}$

***Ans.* $P_u = 33.0$ k** ***Ans.* $P_a = 22.0$ k**

Example 13-10

How many 7/8-in A502, Grade 1 rivets are required for the connection shown in Fig. 13.22 if the plates are A36, standard-size holes are used, and the edge distances and center-to-center distances are 1.5 in and 3 in, respectively? Solve by both the LRFD and ASD methods. For ASD, use the Ω values assumed for Example 13-9. $P_u = 170$ k and $P_a = 120$ k.

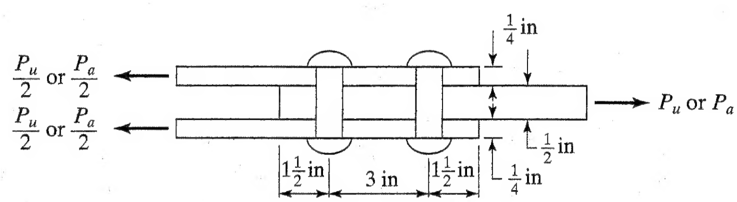


FIGURE 13.22

Solution

Rivets in double shear and bearing on 1/2 in

Nominal double shear strength of 1 rivet

$$r_n = (2A_{rivet})(F_{nv}) = (2 \times 0.6\text{ in}^2)(25.0\text{ ksi}) = 30\text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$	
$\phi r_n = (0.75)(30.0) = 22.5\text{ k}$	$\dfrac{r_n}{\Omega} = \dfrac{30.00}{2.00} = 15.0\text{ k}$	← Controls

Nominal bearing strength of 1 rivet

$$l_c = 1.50 - \frac{\frac{7}{8} + \frac{1}{8}}{2} = 1.00\text{ in}$$

$$r_n = 1.2 l_c t F_u = (1.2)(1.0)\left(\frac{1}{2}\right)(58) = 34.8\text{ k}$$

$$< 2.4\, dtF_u = (2.4)\left(\frac{7}{8}\right)\left(\frac{1}{2}\right)(58) = 60.9\text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi r_n = (0.75)(34.8) = 26.1$ k/rivet	$\frac{r_n}{\Omega} = \frac{34.8}{2.00} = 17.4$ k/rivet

LRFD	ASD
No. of rivets reqd $= \frac{170}{22.5} = 7.56$ **Use eight $\frac{7}{8}$-in A502, grade 1 rivets.**	No of rivets reqd $= \frac{120}{15.0} = 8$ **Use eight $\frac{7}{8}$-in A502, grade 1 rivets.**

Example 13-11

Repeat Example 13-10, using 7/8-in A307 bolts, for which $F_{nv} = 27$ ksi.

Solution. Bolts in double shear and bearing on .5 in:

Nominal shear strength of 1 bolt

$$r_n = (A_{bolt})(F_{nv}) = (2 \times 0.6 \text{ in}^2)(27 \text{ ksi}) = 32.4 \text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$	
$\phi r_n = (0.75)(32.4) = 24.3$ k	$\frac{r_n}{\Omega} = \frac{32.4}{2.00} = 16.2$ k	← Controls

Nominal bearing strength of 1 bolt

$$r_n = 1.2\, l_c t F_u = (1.2)(1.0)\left(\frac{1}{2}\right)(58) = 34.8 \text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi r_n = (0.75)(34.8) = 26.1$ k	$\frac{r_n}{\Omega} = \frac{34.8}{2.00} = 17.4$ k

LRFD	ASD
No. of bolts reqd $= \frac{170}{24.3} = 7.00$ **Use seven $\frac{7}{8}$-in A307 bolts.**	No. of bolts reqd $= \frac{120}{16.2} = 7.41$ **Use eight $\frac{7}{8}$-in A307 bolts.**

13.9 PROBLEMS FOR SOLUTION

For each of the problems listed, the following information is to be used, unless otherwise indicated: (a) A36 steel; (b) standard-size holes; (c) threads of bolts excluded from shear plane.

13-1 to 13-7. *Determine the resultant load on the most stressed bolt in the eccentrically loaded connections shown, using the elastic method.*

13-1. (*Ans.* 23.26 k)

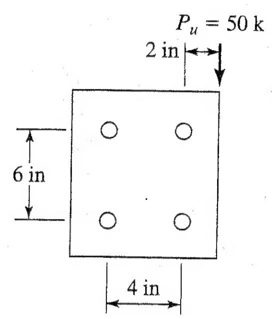


FIGURE P13-1

13-2.

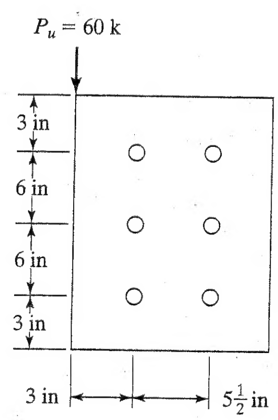


FIGURE P13-2